I M P O R T A N T

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GILES ENGINEERING ASSOCIATES, INC.

Geotechnical Engineering Exploration and Analysis

Proposed Building 111 Addition
Zablocki Veteran Affairs Medical Center
5000 W. National Avenue
Milwaukee, Wisconsin

Prepared for:

Chequamegon Bay Group, Inc.
Wauwatosa, Wisconsin

January 24, 2014
Project No. 1G-1312013

GILES
ENGINEERING ASSOCIATES, INC.

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ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

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January 24, 2014

Chequamegon Bay Group, Inc.
933 North Mayfair Road Suite 320
Wauwatosa, WI 53226

Attention: Mr. Kevin J. Kozak

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Building 111 Addition
Zablocki Veteran Affairs Medical Center
5000 W. National Avenue
Milwaukee, Wisconsin
Project No. 1G-1312013

Dear Mr. Kozak:

Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Exploration and Analysis* for the proposed project. The accompanying report describes the services that were conducted for the project and it provides geotechnical engineering-related findings, conclusions and recommendations that were derived from those services.

We appreciate the opportunity to provide geotechnical engineering consulting services for the proposed project. Please contact the undersigned if there are questions concerning the report or if we may be of further service.

WISCONSIN
JEFFREY S.
MILLER
E-15511
WALES,
WIS.
PROFESSIONAL ENGINEER

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Jeffrey Scott Miller P.E.
Sr. Project Manager

Paul J. Giese, P.E.
Geotechnical Division Manager

Distribution: Chequamegon Bay Group, Inc.
Attn: Mr. Kevin J. Kozak (1 USPS, 1 PDF email: kkozak@cheqbaygrp.com)

N8 W22350 Johnson Drive • Suite A1 • Waukesha, WI 53186
262/544-0118 • Fax 262/549-5868 • E-mail milwauke@gilesengr.com

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PROPOSED BUILDING 111 ADDITION
ZABLOCKI VETERAN AFFAIRS MEDICAL CENTER
5000 W. NATIONAL AVENUE
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PROJECT NO. 1G-1312013

EXECUTIVE SUMMARY

This Executive Summary provides limited geotechnical engineering information regarding the proposed project. Since this Executive Summary is exceedingly abbreviated, it must be read in complete context with the following report.

Subsurface Conditions

- Soils encountered at the test boring locations consist of existing fill and buried topsoil or possible fill from the surface to a depth of 13± feet. Topsoil fill consisting of silty clay trace organic matter was encountered at the surface, and was 15± and 7± inches thick at the locations of Test Boring Nos. 1 and 2, respectively. The existing fill encountered below the topsoil fill consists of silty clay. At Test Boring No. 2, silty clay buried topsoil or possible fill was found at 12± to 13± feet in depth. Underlying soils consist of native, stiff to very stiff consistency silty clay to 23± feet in depth, stiff consistency clayey silt to 38± feet in depth, and stiff to hard consistency silty clay to at least the maximum depth explored of 51± feet at Test Boring No. 1. At Test Boring No. 2, the stiff to hard consistency silty clay extends to 52± feet, underlain with dense sand to 58± feet, and hard consistency silty clay to at least the maximum depth explored of 76± feet.
- It is estimated that the water table was about 23± feet below-grade at the test boring locations, when the Geotechnical Subsurface Exploration Program was conducted.

Foundation Recommendations

- Based on the conditions encountered at the test boring locations, and the assumed floor elevations of the addition, the northern portion of the addition is recommended to be supported with conventionally reinforced continuous wall and column spread foundations below the basement floor slab and founded on suitable bearing native soils. The vestibule portion of the addition is recommended to be supported by drilled shaft foundations founded within the native soils below the unsuitable bearing existing fill and possible fill or buried topsoil. Possible alternate foundation systems for the vestibule portion consist of helical piers and compacted aggregate pier foundations. The canopy addition is recommended to be supported by helical piers designed for both gravity and uplift loads.

Floor Slab Recommendations

- A conventionally reinforced floor slab is recommended for the basement floor of the northern portion addition. A structural floor supported by the drilled shaft foundations or the alternate foundation systems is recommended for the vestibule portion of the addition.

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED BUILDING 111 ADDITION
ZABLOCKI VETERAN AFFAIRS MEDICAL CENTER
5000 W. NATIONAL AVENUE
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1312013

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. (“Giles”) conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained later.

Geotechnical engineering-related recommendations are presented for design and construction of the foundation and floor slab for the additions. Site preparation recommendations are given; however, those recommendations are only preliminary since the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include, but are not limited to, the weather before and during construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development. Environmental consulting was beyond the authorized scope of services for this project.

2.0 SITE DESCRIPTION

The addition is located at the existing Clement J. Zablocki Veterans Affairs Medical Center Building 111 at the address of 5000 West National Avenue, in Milwaukee, Wisconsin. Building 111 is located along the north side of Washington Drive, west of Lincoln Drive. The addition is planned adjacent to the existing south entrance courtyard of the building. The addition is planned to be attached to the east and south building walls of the courtyard area and extend southward to Washington Drive which currently is a lawn landscape area containing a concrete circular driveway and sidewalk at the building entrance. Surface topography descends from the first floor entrance southward to Washington Drive with 3± feet of elevation change at the test boring locations.

3.0 PROJECT DESCRIPTION

The project information provided to Giles consists of Drawing Nos. A100, A101 and A102 each dated December 23, 2013, and Drawing No. C001 dated September 7, 2012. The drawings describe Building 111 as having a basement and is supported by a pile foundation. The first floor and basement floor are described at El. 663.5 and El. 649.7, respectively.

The addition is understood to consist of three portions. The proposed addition location and shape, and existing features of the site are shown on the Test Boring Location Plan, Figure 1 attached with this report, and were adapted from a drawing provided with the project information titled “Soil Boring Locations”, dated December 17, 2013. The northern portion will be attached to the existing Building 111, and will be a one-story building with a basement. The vestibule portion will also be one story but without a basement. An open-air canopy will be located south of the vestibule, and will be used as a cover for patient drop-off and pick-up from vehicles.

The project structural engineer, Mr. Chad Allen told Giles that the structural framing for the northern portion addition will include the eastern exterior wall as a load bearing wall, and a line of load bearing columns offset from the existing building for the western wall of the addition. Also, the vestibule will be supported by load bearing walls, and the canopy will be supported by columns. The project information drawings indicate that the northern portion of the addition will have a first floor at four different levels between El. 663.5 to El. 661.0 and a basement floor at El. 649.8. Also, the canopy pavement is planned at El.658.5 to El. 660.5. The structural loadings were not provided with the project information. For the purposes of this report, the northern portion of the addition is assumed to have a maximum structural live and dead loading of 4± to 6± kips per lineal foot and 125 kips for load bearing walls and columns, respectively, and the basement floor is assumed to have a maximum live loading of 100 psf. The vestibule portion of the addition is assumed to have a maximum structural live and dead loading of 2± to 3± kips per lineal foot, and a first floor live loading of 100 psf. Each of the four columns of the canopy is assumed to have a maximum live and dead gravity loading of 35 kips, and a wind uplift loading of 8 kips.

4.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

The purpose of the Geotechnical Subsurface Exploration Program was to explore the subsurface conditions. Two test borings were drilled approximately at the Client’s suggested locations, and to the approximate depths below grade suggested by the Client. The test borings were drilled on January 2, and January 3, 2014. The test boring locations were positioned on-site relative to existing site features. The approximate test boring locations are shown on the *Test Boring Location Plan* (Figure 1) enclosed in Appendix A.

The ground elevations at the test borings were determined by Giles with a standard level survey relative to the existing building entrance floor temporary benchmark shown on Figure 1 enclosed. The test boring elevations are noted on the *Records of Subsurface Exploration* (enclosed in Appendix A), which are logs of the test borings. The test boring elevations are considered accurate to 1± foot.

Samples were collected from the test borings, at certain depths, using a split-barrel sampler during Standard Penetration Testing (SPT), described in Appendix B, along with descriptions of other field procedures. Immediately after sampling, select portions of the SPT samples were retained in jars that were labeled at the site for identification. The retained samples were transported to Giles' geotechnical laboratory as part of the Geotechnical Subsurface Exploration Program.

5.0 GEOTECHNICAL LABORATORY SERVICES

The retained samples were delivered to our geotechnical laboratory and classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D, and by using the Unified Soil Classification System (ASTM D 2488-75) as a general guide. The classifications are shown on the *Records of Subsurface Exploration*, along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Records of Subsurface Exploration*. For simplicity and abbreviation, terms and symbols are used on the *Records of Subsurface Exploration;* the terms and symbols are defined on the *General Notes*.

Unconfined compression, calibrated penetrometer resistance, vane shear strength, and moisture content tests were performed on select soil samples to evaluate the soils general engineering properties. The test results are on the *Records of Subsurface Exploration*. Laboratory procedures are briefly described in Appendix C.

6.0 MATERIAL CONDITIONS

Since material sampling at the test borings was discontinuous, it was necessary for Giles to estimate conditions between sample intervals. The estimated conditions at the test borings are briefly discussed in this section and are described in more detail on the *Records of Subsurface Exploration*. Also, the conclusions and recommendations in this report are based on the estimated and encountered conditions.

6.1. Surface Materials and Existing Fill

Soils encountered at the test boring locations consist of existing fill and buried topsoil or possible fill from the surface to a depth of 13± feet. Topsoil fill consisting of silty clay trace organic matter was encountered at the surface, and was 15± and 7± inches thick at the locations of Test Boring Nos. 1 and 2, respectively. The existing fill encountered below the topsoil fill consists of silty clay. At Test Boring No. 2, silty clay buried topsoil or possible fill was found at 12± to 13± feet in depth.

6.2. Native Soil

Underlying soils consist of native, stiff to very stiff consistency silty clay to 23± feet in depth, stiff consistency clayey silt to 38± feet in depth, and stiff to hard consistency silty clay to at least the maximum depth explored of 51± feet at Test Boring No. 1. At Test Boring No. 2, the stiff to hard consistency silty clay extends to 52± feet, underlain with dense sand to 58± feet, and hard consistency silty clay to at least the maximum depth explored of 76± feet.

7.0 GROUNDWATER CONDITIONS

It is estimated that the water table was about 23± feet below-grade at the test boring locations, which is El. 639.4± to El. 636.1± when the Geotechnical Subsurface Exploration Program was conducted. Free water was encountered during drilling at a depth of 25± feet below grade at each test boring location. A perched water level may develop in the existing fill at a shallower depth, and groundwater table level conditions will fluctuate depending on the amount of precipitation and water runoff to the site.

The estimated groundwater table depth is only an approximation based on the color of the soil samples retained from the test borings, and water levels that were encountered during the test boring drilling. The actual water table depth may be higher or lower than estimated. Although not considered necessary for this project, if a more precise depth estimate is needed, groundwater observation wells are recommended to be installed and observed at the site. Giles can install and observe the wells, if it is decided that observation wells are necessary.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1. Foundation and Slab Design Considerations

The existing fill encountered at Test Boring Nos. 1 and 2 possibly was placed without engineering controlled conditions. This is based on the black silty clay buried topsoil or possible fill soil encountered between the existing fill and underlying native soils at the location of Test Boring No. 2. Fill placed without engineering control may be composed of variable content and bearing strength soil unsuitable for conventionally reinforced foundations and floor slabs due to intolerable total and differential settlement. Support of foundations and/or floor slabs by the existing fill is not acceptable, according to the project structural engineer, Mr. Chad Allen.

The native soils underlying the existing fill and buried topsoil or possible fill soils are anticipated to be suitable for foundation support of all portions of the addition, and the basement slab for the northern portion of the addition. At Test Boring No. 1 location, the existing fill soils extend to 13± feet in depth, which is at the approximate depth of the planned basement floor elevation for the northern portion of the addition, El. 649.7. Spread foundations immediately below the basement floor of the addition are therefore anticipated to extend to the suitable bearing native soils at the location of Test Boring No. 1. Over excavation to suitable bearing soils is anticipated to be necessary in other areas of the basement, since the existing fill and buried topsoil or possible fill at Test Boring No. 2 was found to extend to 3.7± feet deeper in elevation. The vestibule portion of the addition is planned without a basement, therefore, a drilled shaft and grade beam deep foundation system is recommended, embedded within the underlying suitable bearing native soils. Possible alternate foundation systems for the vestibule portion consist of helical piers and compacted aggregate pier foundations. A structural floor supported by the drilled shaft foundations or the alternate foundation systems is recommended for the vestibule portion of the addition. The canopy addition is recommended to be supported by helical piers designed for both gravity and uplift loads.

Construction expansion joints are recommended for the juncture between the existing building and the planned addition. Also, a construction expansion joint is recommended at the juncture between the northern portion addition, and the vestibule portion addition. The expansion joints are recommended to allow the anticipated differential settlement of the vestibule portion addition supported by a deep foundation system, the northern portion addition with a basement supported by a shallow spread foundation, and the existing building reportedly supported by a pile foundation.

8.2. Seismic Design Considerations

A soil Site Class C is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to a depth of 100 feet below the ground surface. Since 100-foot test borings were not requested or authorized for the project, it was necessary to estimate the Site Class based on the test borings, presumed area geology, and Table 1613.5.2 of the 2006 International Building Code.

8.3. Building Foundation Recommendations

8.3.1. Northern Portion Recommendations

The proposed northern portion of the addition is recommended to be supported by a shallow depth spread foundation system. The spread foundations are recommended to be founded immediately below the planned basement, directly on suitable bearing native soil and/or lean-mix concrete backfill placed continuous from a suitable bearing native soil sub-grade to replace unsuitable bearing soils. The foundations are recommended to be designed using a 3,000 psf maximum, net, allowable soil bearing capacity. Strip footing pads are recommended to be at least 18 inches wide and isolated column pads are recommended to be at least 24 inches wide for geotechnical considerations, regardless of the calculated foundation bearing stress. It is recommended that a structural engineer or architect provide specific foundation details including footing dimensions, reinforcing, and other details.

The recommended spread foundations for support of the northern portion of the addition will subject additional load stress on the soils that surround the pile foundation system reportedly supporting the existing Building 111. The amount of pile settlement caused by the added load stress is anticipated to not cause structural framing distress of the existing building. However the amount of settlement depends on the pile design and pile embedment, which were not provided with the project information. Therefore an analysis of the northern portion addition spread foundation effect on the Building 111 foundation system is recommended to be performed by Giles and the structural engineer or architect.

Spread Footing Embedment Depth Recommendations

It is understood that a minimum 48-inch foundation depth is required by the local building code. Therefore, footings for perimeter walls and other exterior elements of the proposed northern portion of the addition are recommended to bear at least 48 inches below the finished ground grade. The embedment requirement is anticipated to be automatically met for footings directly below the basement floor slab.

It is critical that contractors protect foundation support soil and foundation construction materials such as concrete and reinforcement. In addition, foundation excavations below the basement floor subgrade are recommended to be backfilled with on-site clayey soils placed and compacted as engineered fill in benched excavations immediately after the foundations are capable of supporting lateral pressures from backfill, compaction, and compaction equipment. Earth-forms may be suitable where excavations are extended into the native clayey soils. Formwork may be needed where the soils are not stable, such as within the existing fill, buried topsoil or possible fill.

Spread Foundation Support Soil Requirements

The spread footings are recommended to be directly and entirely supported by suitable-bearing native soil or on lean-mix concrete backfill placed continuous from a suitable bearing native soil subgrade. Based on the recommended 3,000 psf bearing capacity, suitable bearing soils are recommended to have at least a stiff comparative consistency, average unconfined compressive strength value equal to or greater than 1.5 tsf. It is further recommended that the strength characteristics of soil within the entire foundation influence zone (determined by Giles during construction) meet or exceed the recommended values, unless Giles approves lesser depths.

Suitable bearing native soils for direct spread foundation support or for the subgrade of engineered backfill and indirect foundation support are anticipated to be present at the depths and elevations shown on the table below.

TABLE 1 ANTICIPATED SUITABLE BEARING GRADE DEPTH AND ELEVATION FOR DIRECT/INDIRECT FOUNDATION SUPPORT (1)		
Test Boring Location	Anticipated Suitable Bearing Grade	
	Depth Below Existing Surface (2) (feet)	Elevation (3)
1	13±	649.4±
2	13±	646.1±

1. Maximum net, allowable bearing capacity of 3,000 psf
2. Depth below the approximate existing ground surface at the time of drilling on January 2 and 3, 2014.
3. Referenced to the elevations determined by Giles relative to the Building 111 first floor temporary benchmark shown on Figure 1 enclosed in Appendix A.

For design and construction estimating purposes, the suitable bearing grade may be interpreted linearly between the test boring locations. The actual suitable bearing grade may differ because the subsurface conditions may differ beyond the test boring locations; as such, the geotechnical engineering recommendations in this report are predicated upon Giles evaluating the suitability of the foundation support soils during construction to check that the foundations are supported within and underlain by suitable bearing materials.

It is recommended that Giles evaluate foundation support soil immediately before foundation construction. The purpose of the recommended evaluation is to confirm that the foundation will be properly supported and confirm that the support soil is similar to the conditions described on the *Records of Subsurface Exploration*. In the event that another firm performs the recommended foundation evaluation of foundation support soil, they should use appropriate means and methods and Giles must be notified if the composition or strength characteristics of foundation support soil differ from those shown on the *Records of Subsurface Exploration* so that alterations to our recommendations can be made if needed.

Soil that is within the foundation influence zone but does not meet the recommended strength criteria, or is otherwise unsuitable, is recommended to be replaced. Unsuitable bearing material is recommended to be replaced with lean-mix Portland cement concrete backfill with a minimum 28-day compressive strength of 500 psi. Giles can provide recommendations pertaining to soil over-excavation and replacement at the time of construction. As an option to soil replacement, strip footing pads could be stepped or thickened to extend through unsuitable bearing materials and isolated column pads could be uniformly thickened. It is recommended that a structural engineer or architect provide the specific details of stepped or thickened footings.

Estimated Spread Foundation Settlement

The post-construction total and differential settlements of a spread footing foundation designed and constructed based on the recommendations of this report are estimated to be less than about 1.0 inch and 0.5 inch, respectively. The post-construction angular distortion is estimated to be less than about 0.0021 inch per inch across a distance of 20 feet or more. Estimated settlements are based on the assumption that soils will be thoroughly tested and approved by a licensed qualified geotechnical engineer during foundation construction. The estimated settlements are considered within tolerable limits for the planned development provided they are properly considered in the architects and structural engineers design.

8.3.2. Vestibule Portion Recommendations

The walls and the structural floor slab of the vestibule portion of the addition are recommended to be supported by straight-shaft drilled piers without enlarged bases or bells, designed for shaft friction and end bearing. The drilled pier shafts and bottoms are recommended to be embedded into the suitable bearing native soils. The required embedment depth into suitable bearing native soil will depend on the structural loads and the shaft diameter. A higher compression capacity is generally available with larger diameter shafts, shafts drilled to deeper depths, or capacities based on using in-situ pressuremeter or load tests.

Possible alternate foundation systems for the vestibule portion consist of helical piers, or compacted aggregate pier foundations. Recommendations for a helical pier foundation system are presented in this report, **Section 8.3.3 Canopy Recommendations**. The design of compacted aggregate piers requires the assistance of a specialty contractor; therefore, recommendations for compacted aggregate piers can be provided in an addendum, if desired.

Drilled Shaft Foundation Support Soil Requirements

Suitable soils along the shaft and to at least a depth of one shaft diameter below the shaft base are recommended to have at least a dense relative density, average, corrected, N-value equal to or greater than 20 (determined from SPTs and correlated from other in-situ tests) for non-cohesive sandy foundation support soil. For cohesive silty clay and clayey silt foundation support soil, suitable soils along the shaft and to at least a depth of one shaft diameter below the shaft base are recommended to have at least a stiff comparative consistency, average unconfined compressive strength equal to or greater than 1.5 tons per square foot (tsf). It is recommended that the strength characteristics of soil within the above-described zone meet or exceed the recommended values, unless Giles approves lesser depths.

The estimated depths of embedment and the diameter of the drilled shafts at the test boring locations are presented in the table below for the wall load of 2± to 3± kips per lineal foot plus the structural floor slab load of 4± kips per lineal foot, a total of 7± kips per lineal foot assumed by Giles. Estimated allowable loads for other drilled shaft diameters or embedment can also be provided in an addendum report, if desired. A safety factor of 3 was used in estimating the allowable capacity.

TABLE 2 VESTIBULE PORTION ADDITION ALLOWABLE CAPACITY PER DRILLED SHAFT			
Allowable Capacity (Kips) (1)	Diameter of Straight Shaft (feet)	Embedment Depth(2) (feet)	Embedment Elevation
40	2.5	49±(3) / 54±(4)	613.4±(3) / 605.1±(4)
50	3	49±(3) / 54±(4)	613.4±(3) / 605.1±(4)
65	3.5	49±(3) / 54±(4)	613.4±(3) / 605.1±(4)

1. A factor of safety of 3 was applied.
2. Depth below the approximate existing ground surface at Test Boring Nos. 1 and 2 at the time of drilling on January 2 and 3, 2014.
3. Depth below surface or elevation at Test Boring No. 1.
4. Depth below surface or elevation at Test Boring No. 2.

The suitable bearing grade may be interpolated linearly between the test boring locations for design and construction estimating purposes. The actual suitable bearing grade may differ because varying soil conditions were encountered at the test boring locations and the subsurface conditions may differ beyond the test boring locations; as such, the geotechnical engineering recommendations in this report are predicated upon Giles evaluating the suitability of the foundation support soils during construction to check that the foundations are supported within and underlain by suitable bearing materials. If unsuitable bearing soils are encountered during drilled shaft excavation, extension of the drilled shaft deeper into suitable bearing native soils or possibly a redesign of the drilled shaft is recommended. The depth to actual suitable bearing strength soils within the drilled shaft may vary from that estimated at the test boring locations.

Estimated Drilled Shaft Foundation Settlement

Post construction total and differential settlements for a drilled shaft foundation system designed in accordance with the above recommendations supported by suitable bearing native soils are estimated to be less than 0.75 and 0.4 inches, respectively, with an angular distortion of less than 0.0017 inch per inch for a span of 20 feet or more.

General Drilled Shaft Construction Recommendations

It is understood that a minimum 48-inch foundation depth is required by the local building code. Therefore, the grade beams for the drilled shaft or compacted aggregate piers for perimeter walls and other exterior elements of the proposed northern portion of the addition are recommended to extend in depth at least 48 inches below the finished ground grade.

Concrete should consist of a Portland cement mixture properly air-entrained with an appropriate water/cement ratio for proper strength and durability. The concrete may be placed by either free-falling or by being pumped in the shaft; however, the concrete slump and maximum aggregate size must be selected so that the concrete will flow easily between reinforcing bars and will completely fill all voids. Care must be taken so that the concrete will not contact the reinforcing cages during placement otherwise segregation of the concrete will occur. If the mixture is too stiff, reinforcing cages may be pulled up when liners are withdrawn.

Due to the fill and layers/lenses of granular soils, excavation difficulties are expected to be encountered due to caving of shaft sidewalls. Steel liners or casing are expected to be necessary in the drilled shafts at the time of drilling to maintain an open excavation. Liners can be removed as concrete is poured maintaining several feet of concrete head above the bottom of the casing. Prior to placement of the concrete, the bottoms of the drilled shafts must be observed for cleanliness, that the drilled shaft bottom soil is relatively undisturbed, and that dimensions are correct. The concrete must be placed in accordance with "state-of-the-practice" procedures under engineering controlled conditions as noted below.

Drilled shafts should have a minimum shaft diameter of 24-inches to help eliminate arching and the development of possible voids in placement of the concrete. Minimum on-center spacing should be two times the shaft diameter plus 5 feet at the bearing surface to reduce the overlapping stress influence. Drilled shaft construction should be done in accordance with American Concrete Institute documents (ACI 336.1R-98 and ACI 336.3 R-93).

Drilled shaft excavations should not be allowed to stand open for any significant length of time since this may allow water to accumulate. A clean-out bucket should be provided to permit manual removal of all loose or disturbed soils within the drilled shaft excavation.

It is estimated that the water table and/or the perched water level was 23± feet below-grade at the test borings when the Geotechnical Subsurface Exploration Program was conducted. Special dewatering methods may be necessary during drilled shaft construction. No more than 3 inches of water may remain in drilled shaft excavation bottoms when concrete is placed. Where water is encountered, it is expected to be necessary to use "drilling mud", downhole dewatering or bentonite slurry and tremie concrete construction methods.

Giles does not recommend downhole inspection for safety reasons. If downhole inspection is required, care must be taken to ensure that drilled shafts are cased and tested for possible accumulation of "bad air" or toxic gas prior to permitting an individual to proceed down the hole. Carbon Dioxide (CO_2) or Carbon Monoxide (CO) gases, being heavier than air, may accumulate in the shaft from internal combustion engines, such as pumps, air compressors and/or vehicular traffic; or may be present within the subsoil.

8.3.3. Canopy Recommendations

The four columns of the canopy are recommended to be supported by helical piers extended through the existing fill and the buried topsoil or possible fill, and embedded into the suitable bearing native soils. The required embedment depth into suitable bearing native soil will depend on the structural loads and the helical pier size and number of helices configuration.

Estimated downward (compression) capacities for various sizes of helical piers are provided in the following Table 3, and estimated uplift (tension) capacities for various sizes of helical piers are provided in the following Table 4. The helical piers are recommended to extend to a depth such that the upper helix is at least 5 feet into suitable bearing native soil.

TABLE 3 –ALLOWABLE DOWNWARD COMPRESSION LOAD PER HELICAL PIER

Helical Pier Tip Depth (1) (feet)	Helical Pier Tip Elevation (2)	Number of Helix Plates	Diameter of Helix Plates (inches)	Estimated Allowable Capacity[3] (kips)
24	El. 635±	3	6, 8, 10	4
24	El. 635±	3	10, 12 and 14	10

1. Depth below the approximate existing ground surface at Test Boring No. 2 at the time of drilling on January 3, 2014.
2. Tip elevation is based on 7 feet lead shaft length and the recommended minimum upper helix embedment depth of 5 feet into the bearing layer soil anticipated at El. 646.
3. Downward (compression) Capacity. Factor of Safety of 2 applied.

TABLE 4 –ALLOWABLE UPLIFT TENSION LOAD PER HELICAL PIER

Helical Pier Tip Depth (1) (feet)	Helical Pier Tip Elevation (2)	Number of Helix Plates	Diameter of Helix Plates (inches)	Estimated Allowable Capacity[3] (kips)
24	El. 635±	3	6, 8, 10	4
24	El. 635±	3	10, 12 and 14	10

4. Depth below the approximate existing ground surface at Test Boring No. 2 at the time of drilling on January 3, 2014.
5. Tip elevation is based on 7 feet lead shaft length and the recommended minimum upper helix embedment depth of 5 feet into the bearing layer soil anticipated at El. 646.
6. Downward (compression) Capacity. Factor of Safety of 2 applied.

A higher capacity is generally available with a larger diameter helix, multiple helices or helical piers installed to deeper depths. Other sizes may be available and selected by the contractor, and design recommendations can be provided by Giles in an addendum report, if needed. The helical pier cap design is recommended to be performed by the structural engineer or architect. Varying soil conditions were encountered at the test borings. Therefore, drilling the helical piers to deeper embedment depths may be needed in areas to reach the desired capacity.

The vertical helix separation distance for an individual pier should be at least 300 percent the diameter of the next lowest helix. Generally, the minimum horizontal center to center spacing between helical piers is 3± to 4± feet for construction installation clearances. Helical piers are recommended to have a minimum shaft diameter of 2.75 inches and a minimum wall thickness of at least 0.250 inches.

It is recommended the helical pier installer evaluate the actual capacity of each vertical steel helical pier by monitoring the installation torque at or below the anticipated suitable depth and elevation noted above. The minimum installation torque for suitable vertical capacity is recommended to be correlated to an ultimate vertical capacity of at least 200% of the allowable capacity shown in Tables 3 and 4, above. A torque factor of [8 ft^{-1}] is recommended to be used to determine the correlated pier capacity, based on a helical pier with a 2.75-inch diameter shaft. A reduced torque factor will be needed if helical piers with larger shaft diameters are used. The torque testing is recommended to be performed by the pier contractor, using the ultimate vertical capacity determined from installation torque and the recommended torque factor. Observation and testing is recommended to be performed by *Giles* so that the foundations are supported within suitable bearing soils.

<u>Helical Pier Installation Considerations</u>

Although rubble was not encountered in the test borings, installation difficulties for helical piers may occur due to rubble or other variable materials, considering the presence of the existing fill soils. Pre-drilling, prior to helical pier installation, may be needed to prevent damage to the helical piers and/or permit helical pier installation to the required depths. Pre-drilling, where performed, is recommended to be limited to within the existing fill soils and pre-drilling should in no case extend deeper than a depth equivalent to 3 helix diameters above the uppermost helix.

<u>Estimated Helical Pier Settlement</u>

The post-construction total and differential settlements of a helical pier foundation system designed and constructed based on the recommendations of this report are estimated to be less than approximately 1.0 inch and 0.5 inch, respectively. The post-construction angular distortion is estimated to be less than 0.002 across a distance of 20 feet or more.

8.4. Floor Slab Recommendations

The basement floor slab for the northern portion of the addition may consist of a ground-supported, concrete slab. The at-grade floor slab for the vestibule portion of the addition is recommended to consist of structural floor slab supported by the drilled shaft foundation system.

8.4.1. Northern Portion Addition Basement Floor Slab

With proper subgrade preparation, it is anticipated that the native clay soils encountered at the location of Test Boring No. 1 will be suitable for a ground-supported, basement floor slab. Over-excavation of unsuitable bearing existing fill, buried topsoil or possible fill such as found at Test Boring No. 2 to El. 646.1± and replacement with engineered fill may be necessary to develop a suitable floor slab sub-grade considering the strength characteristics of the existing fill.

The ground-supported, floating, and isolated at-grade concrete slab may be designed for a modulus of subgrade reaction, K_{v1}, of 175 pounds per cubic inch; which represents a 1 foot by 1 foot square plate modulus value. The floor slab is recommended to be reinforced with welded wire fabric to help control shrinkage cracking. In lieu of welded wire fabric, the floor slab concrete could alternatively contain an appropriate concrete admixture, such as fiber mesh to help control shrinkage cracking. It is recommended that a structural engineer or architect specify the floor slab thickness, reinforcing, joint details and other parameters. Base course recommendations are provided below.

Estimated Basement Floor Slab Settlement

The post-construction total and differential settlements of the floor slab constructed in accordance with the recommendations of this report are estimated to be less than about 0.5 inch and 0.3 inch, respectively, over a distance of about 20 feet. Estimated settlements are based on the assumption that the subgrade soils will be tested and approved by Giles during slab construction. The estimated settlements are considered within tolerable limits for the planned development provided they are properly considered in the architects and structural engineers design.

8.4.2. Vestibule Portion Addition Floor Slab

The floor of the vestibule portion of the addition is recommended to be a structural floor slab supported by the drilled shaft foundation system.

8.5. **Basement Below-Grade Wall Recommendations**

For the northern portion of the addition, a drainage system is recommended to surround the below-grade basement walls and be connected to a suitable drainage facility. Also, the below-grade walls should be designed to withstand earth pressures and lateral pressures from surcharges near the walls. Drainage system recommendations and geotechnical design parameters for below-grade walls are provided below.

Drainage System Recommendations

A drainage system is recommended to remove water near the below-grade walls. It will lessen the potential for water pressure build-up against the below-grade walls, which could cause wall movement, wall distress, and interior water or moisture problems. The drainage system is recommended to use drainage aggregate backfill surrounded with a geotextile filter and separator, or is recommended to consist of a drainage geocomposite, with either alternate connected to the drainpipe system for drainage.

Continuous drainpipes should be installed at the same elevation along the interior and exterior sides of the perimeter strip footing pads; creating interior and exterior drainage loops. The drain pipes should be minimum 4-inch-diameter perforated pipes specifically designed for drainage applications. The interior and exterior drain pipe systems should connect to a central sump crock or crocks, or connected to the storm sewer system for removal. Drain pipe bleeders should be cast in the perimeter strip footing pads to serve as water conduits between interior and exterior drainpipes. The bleeder pipes should be about 3 inches in diameter and about 6 to 8 feet-on-center. Proper damp-proofing should also be applied to the exterior walls.

The drainage aggregate alternate is recommended to be in accordance with the WDOT Standard Specifications, Section 501.3.6.4.5 Size No.1, sometimes locally known as crushed, clear, No.1 stone. The geotextile filter and separator is recommended to consist of a non-woven geotextile in accordance with the WDOT Standard Specifications, Section 645, Type DF, Schedule A or B. The drainage geocomposite alternate is recommended to consist of TENAX TENFLOW 770-2, or alternate drainage geocomposite approved for use by Giles prior to installation.

Drainage aggregate or the drainage geocomposite should abut the below-grade walls as part of the drainage system. The aggregate or the drainage geocomposite will serve as drainage media against the below-grade walls. The aggregate layer should be at least 2 feet wide, measured horizontally from the below-grade walls. Also, the bottom of the aggregate layer or the drainage geocomposite is recommended to be at the same elevation as the bottom of the basement floor slab drainage aggregate base course and it should continuously abut the below-grade walls.

Backfill that is placed adjacent to below-grade walls, and will also provide structural support, should be compacted in accordance with the *Guide Specifications* enclosed. Compaction should be performed to between 92 and 95 percent of the maximum dry density obtained by the Standard Proctor compaction test (ASTM D698) or to an in-place density specified by the project structural engineer. The drainage aggregate should be compacted in maximum 8 to 12-inch-thick lifts (measured loose). Heavy compaction equipment, such as mechanical rollers, should not operate within about 10 feet of the below-grade walls because high lateral pressures could develop and the walls could move and possibly fail. Hand-operated compaction equipment, such as vibratory plates, should be used within about 10 feet of below-grade walls. Below-grade walls should be braced during construction, backfilling, and compaction. The bracing should remain in-place until the below-grade floor slab and main deck are installed.

Lateral Pressure Design Parameters

A structural engineer should design the below-grade walls to resist lateral pressures from the adjacent soil and any surface surcharges. Assuming that aggregate will continuously abut the below-grade walls as recommended, an equivalent "at-rest" fluid pressure of 75 psf per foot of depth may be used for below-grade wall design. If the recommended drainage system is not installed along the below-grade walls and soil that is not free-draining, such as silty clay, abuts the below-grade walls, a higher lateral pressure, likely in the range of 90 to 100 psf per foot depth, may develop. Silty or clayey soil should not be within about two feet of the below-grade walls due to high earth pressures, potential frost damage, and insufficient drainage, which could cause the below-grade walls to become damp or wet. Lateral pressures caused by any surcharge loads should be added to the "at-rest" fluid pressure recommended above. We can provide supplemental recommendations regarding surcharge loads on a case-by-case basis. Below-grade walls that are not designed to resist the actual pressures will be prone to dampness and/or lateral movement and potential distress.

Cast-in-place Portland cement concrete may be used to construct the below-grade walls. Basement wall construction and reinforcing should be, at a minimum, in accordance with Chapter 18 of the 2006 International Building Code (IBC). Wall design and construction must include adequate reinforcing to resist lateral pressures.

Basement Slab Base Course Recommendations

A minimum 10-inch-thick base course is recommended to be directly below the basement floor slab to serve as a capillary break, provide more uniform floor slab support and serve as a drainage layer. It is recommended that the base course consist of free-draining aggregate. Also, it is recommended that *Giles* test and approve base course aggregate before it is placed. Depending on aggregate gradation, a non-woven geotextile might need to be below the base course.

A minimum 10-mil vapor retarder is recommended to be directly below the floor slab or base course throughout the entire floor area. It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture on future flooring materials. The vapor retarder is recommended to be in accordance with ASTM E 1745-97, which is entitled:

Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs. If the base course has sharp, angular aggregate, protecting the retarder with a geotextile (or by other means) is recommended.

8.6. Generalized Site Preparation Recommendations

The means and methods of site preparation will greatly depend on the weather conditions before and during construction, the subsurface conditions that are exposed during earthwork operations, and the finalized details of the proposed development. Therefore, only generalized site preparation recommendations are given.

In addition to being generalized, the following site preparation recommendations are abbreviated; the *Guide Specifications* in Appendix D gives further recommendations. The *Guide Specifications* should be read along with this section. Also, the *Guide Specifications* are recommended to be used as an aid to develop the project specifications.

Clearing, Grubbing and Stripping

Site preparation will require complete removal and proper disposal of the existing driveway pavement materials, all surface vegetation, and the topsoil fill found at the surface of the site when the test borings were drilled on January 2 and 3, 2014. The existing concrete pavement surface could be pulverized into a maximum 3-inch sized material and stockpiled on-site for use as fill, or sub-grade stabilization material. The existing base course material, if any, may be reused as a fill material. The topsoil fill at the surface was found to extend to 15± inches and 7± inches in depth at Test Boring Nos. 1 and 2, respectively. The topsoil fill depth recommended to be stripped from the site may vary between and beyond the test boring locations.

Proof-Rolling and Fill Placement

After the recommended clearing, grubbing, and stripping, and after any site cut is completed, the subgrade is recommended to be proof-rolled with a fully-loaded, tandem-axle dump truck or other suitable construction equipment to help locate unstable soil based on subgrade deflection caused by the wheel loads of the proof-roll equipment. It is recommended that Giles observe proof-roll operations and evaluate the sub-grade stability based on those observations.

Soil that yields excessively or ruts during proof-rolling, or shows other signs of instability, is recommended to be replaced with engineered fill. As an option to replacement, unsuitable soil could be scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned (uniformly moistened or dried), and compacted to the required in-place density. Unsuitable soil could also be modified with hydrated lime or Portland cement, or mechanically stabilized with coarse aggregate and/or with geogrids or geotextiles. It is recommended that soil improvement recommendations be provided by Giles based on the conditions during construction.

The site is recommended to be raised, where necessary, to the planned finished grade with engineered fill immediately after the sub-grade is confirmed to be stable and suitable to support the proposed site improvements. Engineered fill is recommended to be placed in uniform, relatively thin layers with each layer recommended to be compacted to at least 95 percent of the fill material's maximum dry density determined from the geotechnical test titled: *Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort* (ASTM D698). That test is hereafter referenced as: *The Standard Proctor Compaction test*.

The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, as described in Item No. 5 of the *Guide Specifications*. The optimum moisture content is to be determined by the Standard Proctor Compaction test.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after Giles confirms that the previous lift was properly placed and compacted. Sub-grade soil may need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

Use of Site Soil as Engineered Fill

Soils excavated from the site are considered unsuitable for use beneath the spread foundations and floor slab recommended for the basement of the north portion of the addition. Aggregate fill material is recommended to be imported to the site for use beneath the foundations and floor slab. The soils excavated from the site may be used as engineered fill beneath the structural floor slab of the vestibule portion of the addition and pavements. Additional recommendations regarding fill selection, placement and compaction are given in the *Guide Specifications*.

8.7. Generalized Construction Considerations

Adverse Weather

Site soil is moisture sensitive and will become unstable when exposed to adverse weather such as rain, snow, and freezing temperatures. Therefore, it might be necessary to remove or stabilize the upper 6 to 12 inches (or more) of soil due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation and/or stabilization of unstable soil should be expected if construction is during or after adverse weather. Some over-excavation is expected to be needed even if construction is during favorable, dry weather due to the existing fill. Because site preparation is weather dependant, bids for site preparation, and other earthwork activities, should consider the time of year that construction will be conducted.

In an effort to protect soil from adverse weather, the site surface is recommended to be smoothly graded and contoured during construction to divert surface water away from construction areas. Also, contoured sub-grades are recommended to be rolled with a smooth-drum compactor, before precipitation, to “seal” the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas in an effort to reduce traffic-related soil disturbance.

Dewatering

It is estimated that the water table was about 23± feet below-grade at the test borings when the Geotechnical Subsurface Exploration Program was conducted on January 2 and 3, 2014. Perched water conditions may also be present at shallower depths at the time of construction. Excavations for the basement foundations are anticipated to be above the estimated water table level, but the drilled shaft or compacted aggregate piers recommended for support of the vestibule portion of the addition probably will encounter ground water and or perched water levels, and dewatering during drilled shaft or compacted aggregate pier foundation construction should be expected. Additional recommendations for dewatering drilled pier construction excavations are presented in Section 8.3 of this report.

Excavation Stability and Considerations

Stability and caving problems may be encountered in excavations for utility conduits, foundations, foundation grade beams, and the drilled shaft or compacted aggregate pier construction, due to the strength characteristics of the variable fill materials, and underlying native soils. Excavations are recommended to be made in accordance with current OSHA excavation and trench safety standards, and other applicable requirements. Sides of excavations might need to be sloped or braced to maintain or develop a safe work environment. Temporary shoring must be designed according to applicable regulatory requirements. Contractors are responsible for excavation safety.

Precautions must be taken so that excavations for the new construction do not undermine existing footings and floor slabs or otherwise compromise the existing structure support. Depending on the actual details of the existing building and proximity of the new foundation excavations to the existing building foundations, underpinning may be needed. If voids occur below existing footings or floor slab, Giles should be contacted to observe the conditions and provide recommendations. In general, voids should be immediately filled with a concrete dry pack or injection of a non-shrink expansive sand and cement slurry under appropriate pressure to maintain contact between the structure and supporting soils.

Existing Fill Considerations

Existing fill was encountered at the test boring locations. Considering the existing fill and existing construction, unsuitable bearing materials may have been buried beneath the site surface during previous grading and/or construction. Questionable materials, where encountered, are recommended to be evaluated by Giles to determine if removal and replacement with engineered fill is necessary. Disposal of any unsuitable material should be in accordance with local, state and federal regulations for the material type. Alteration to the recommendations of this report may be needed, if conditions different than those noted on the *Records of Subsurface Exploration* are revealed.

Existing Utilities

All existing utilities are recommended to be located, and any planned to be maintained should be relocated outside the proposed building addition areas, if possible. Utilities that are not reused should be capped-off and removed or properly abandoned in-place in accordance with local codes and ordinances. The excavations for utilities to be removed that are in the influence zone of new construction are recommended to be backfilled with engineered fill placed under engineering controlled conditions. Underground utilities that are to be reused or abandoned in-place should be evaluated by the plumbing contractor, and utility backfill should be evaluated by Giles, to determine their potential effect on the new development. Grading operations must be done carefully so that existing utilities are not damaged or disturbed. Utility invert elevations, depths and sizes should be checked relative to the planned foundation elevations to determine what specific concerns are present.

8.8. Recommended Construction Materials Testing Services

This report was prepared assuming that Giles will perform Construction Materials Testing ("CMT") services during construction of the proposed development. In general, CMT services are recommended to at least include observation and testing of: foundation and floor slab support soil; concrete; asphalt, and other construction materials. It might be necessary for Giles to provide supplemental geotechnical recommendations based on the results of CMT services and specific details of the project not known at this time.

9.0 BASIS OF REPORT

This report is based on Giles' Proposal No. 1GP-1203028, dated March 12, 2012. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

This report is based on the project description given earlier in this report. Giles must be notified if any parts of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Records of Subsurface Exploration* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

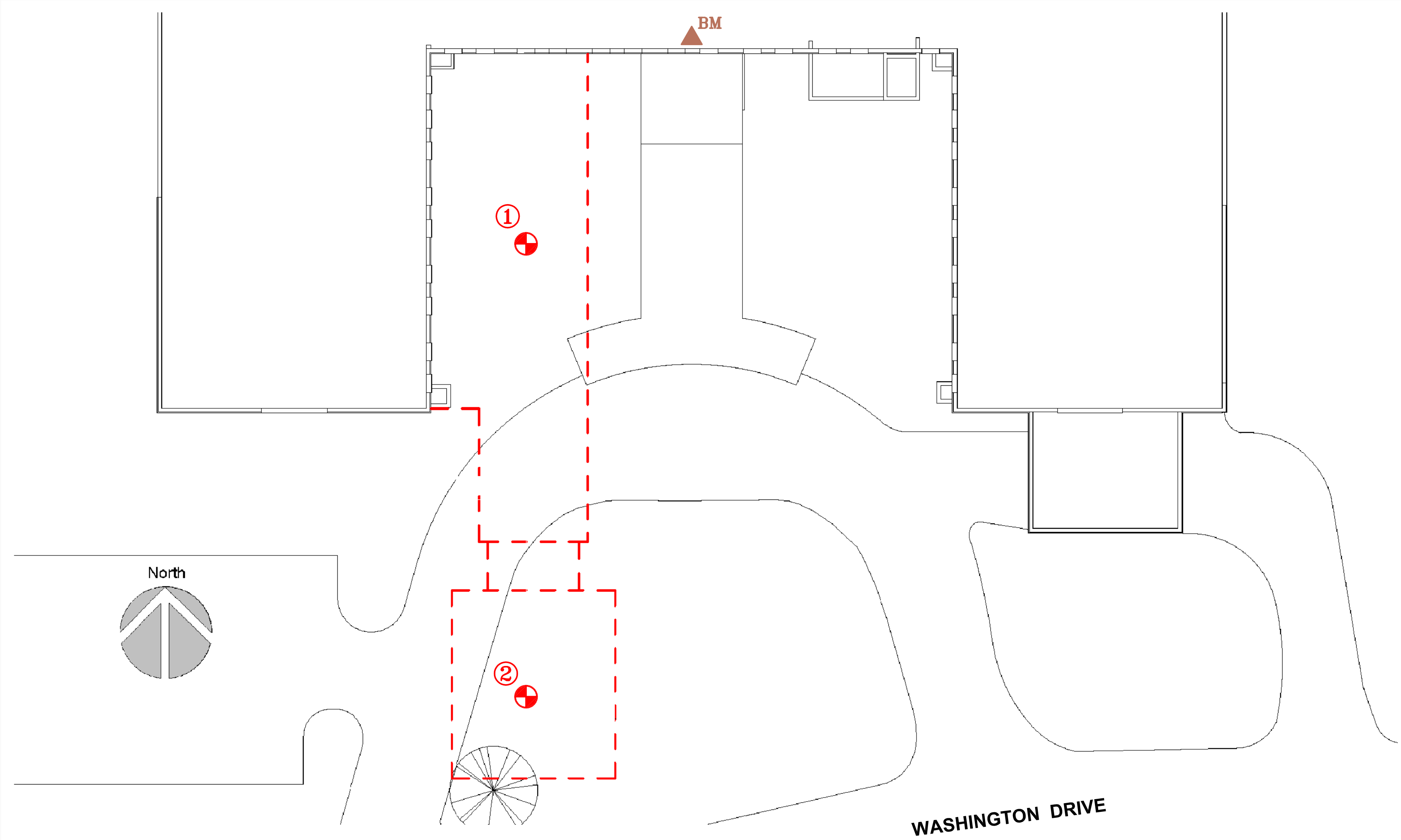
1G-1312013-report/13Geo04/jsm/ldk

APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DRIVE, SUITE A1
WAUKESHA, WI 53186 (262)544-0118

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED BUILDING 111 ADDITION
ZABLOCKI VA CENTER
5000 W. NATIONAL AVENUE
MILWAUKEE, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
JSM	JSZ	approx. 1"=40'	01-16-14	--
PROJECT NO.: 1G-1312013			CAD No. 1g1312013-blp	

LEGEND:

1 GEOTECHNICAL TEST BORING

BM BENCHMARK: FIRST FLOOR SLAB SURFACE AT MAIN ENTRANCE. ASSUMED ELEVATION = 663.0'

0 20' 40'

APPROXIMATE SCALE

NOTES:
1.) TEST BORING LOCATIONS ARE APPROXIMATE.
2.) BASE MAP DEVELOPED FROM THE "SOIL BORING LOCATIONS", DATED 12-17-13, PROVIDED BY THE CLIENT.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 1	PROJECT: Proposed Building 111 Addition
SURFACE ELEVATION: 662.4	PROJECT LOCATION: 5000 W. National Avenue
COMPLETION DATE: 1/2/14	Milwaukee, Wisconsin
FIELD REPRESENTATIVE: Ryan Fett	GILES PROJECT NUMBER: 1G-1312013

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles
Dallas Atlanta
Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
15"± Black Silty Clay, trace Organic Matter (Topsoil Fill)-Moist		1-SS	10		3.7		17		(a)
Gray-Brown Silty Clay, trace fine Sand (Fill)-Damp		2-SS	12	2.9	2.7		16		
	5	3-SS	11	2.2	2.7		16		
		4-SS	10						(b)
	10	5-SS	20		2.5		16		(c)
Brown slightly Orange-Brown mottled Silty Clay, trace fine to coarse Sand and Gravel (contains Brown Silty fine Sand lenses or seams at 18± feet)-Damp	15	6-SS	22	3.2	4.2		16		
		7-SS	31				9		
Moist at 20± feet	20	8-SS	12	3.6	3.5		23		
Gray Clayey Silt, little to some fine Sand-Moist to Wet	▽ 25	9-SS	14	2.7	2.2		14		
Gray Clayey SIlt, trace fine Sand-Moist to Wet	30	10-SS	12	1.1	1.0	0.26	18		
	35	11-SS	15	2.9	2.7		17		
Gray SIlty Clay, trace fine Sand-Moist	40	12-SS	14	3.7	3.0		13		
	45	13-SS	12	2.8	2.2		16		
	50	14-SS	47	7.8	4.5+		14		

Boring Terminated at 51 Feet

WATER OBSERVATION DATA	REMARKS
▽ WATER ENCOUNTERED DURING DRILLING: 24.5 ft.	(a) Frozen to 1± foot in depth.
▼ WATER LEVEL AFTER REMOVAL: None	(b) Sample 4-SS; no recovery, auger sample obtained.
CAVE DEPTH AFTER REMOVAL: 45.0 ft.	(c) Sample 5-SS; poor sample recovery.
▼ WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test borings is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: 2	PROJECT: Proposed Building 111 Addition	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 659.1	PROJECT LOCATION: 5000 W. National Avenue	Milwaukee Los Angeles
COMPLETION DATE: 1/3/14	Milwaukee, Wisconsin	Dallas Atlanta
FIELD REPRESENTATIVE: Ryan Fett	GILES PROJECT NUMBER: 1G-1312013	Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
7"± Black Silty Clay, trace Organic Matter (Topsoil Fill)-Moist		1-SS	13	1.7	4.5+		12		(a)
Gray-Brown Silty Clay, little fine Sand (Fill)-Moist		2-SS	14	8.6	4.5+		15		
Brown Silty Clay, trace fine Sand (Fill)-Moist	5	3-SS	17	5.7	4.5+		19		
Brown and Gray Silty Clay, little fine Sand (Fill)-Damp		4-SS	14				16		
Brown Silty Clay, trace fine Sand (Fill)-Damp	10	5-SS	11		4.5+		17		
Black Silty Clay-Damp (Buried Topsoil or Possible Fill)		6-SS	11	4.7	4.5+ 4.5		25 26		
Brown slightly Orange-Brown mottled Silty Clay-Damp Moist at 15± feet	15	7-SS	8	1.6	1.7		23		
Brown to Gray Silty Clay-Moist (contains Brown Silty fine Sand Seams or Lenses)	20	8-SS	18	3.3	4.0		19		
Gray Clayey Silt, little to some fine Sand-Moist to Wet	▽ 25	9-SS	12	1.7	2.0		16		
	30	10-SS	11						(b)
Gray Clayey Silt, trace fine Sand-Moist to Wet	35	11-SS	8	0.7	0.7		16		
Gray Silty Clay, trace fine Sand-Moist	▼ 40	12-SS	9	1.7	1.5		15		
	45	13-SS	22	6.6	4.5+		17		

WATER OBSERVATION DATA	REMARKS
▽ WATER ENCOUNTERED DURING DRILLING: 25.0 ft.	(a) Frozen to ½± foot in depth.
▼ WATER LEVEL AFTER REMOVAL: 40.0 ft.	(b) Sample 10-SS; no recovery.
CAVE DEPTH AFTER REMOVAL: 52.0 ft.	
▼ WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test borings is shown on the Boring Location Plan.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION:	PROJECT:
2	Proposed Building 111 Addition
SURFACE ELEVATION:	PROJECT LOCATION:
659.1	5000 W. National Avenue
COMPLETION DATE:	
1/3/14	Milwaukee, Wisconsin
FIELD REPRESENTATIVE:	
Ryan Fett	GILES PROJECT NUMBER: 1G-1312013

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Los Angeles Dallas Atlanta Washington, D.C. Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Gray Silty Clay, trace fine Sand-Moist *(continued)*		14-SS	36	8.4	4.5+		12		
Gray Silty fine Sand, trace Clay-Moist to Wet	55	15-SS	55						
Gray Silty Clay-Moist (contains thin Silt seams and lenses)	60	16-SS	23		4.5+		17		
Gray Silty Clay-Moist	65	17-SS	44	6.6	4.5		16		
	70	18-SS	19	4.5	4.0		20		
	75	19-SS	20	2.2	2.0		19		

Boring Terminated at 76 Feet

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: 25.0 ft.	(a) Frozen to ½± foot in depth.
WATER LEVEL AFTER REMOVAL: 40.0 ft.	(b) Sample 10-SS; no recovery.
CAVE DEPTH AFTER REMOVAL: 52.0 ft.	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

NORMAL BORING LOGS 1G1312013.GPJ GIL_CORP.GDT 1/24/14

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test borings is shown on the Boring Location Plan.

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D 420 entitled “Standard Guide for Sampling Rock and Rock” and/or other relevant specifications. Soil samples were preserved and transported to *Giles’* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled “Standard Practice for Preserving and Transporting Soil Samples.” Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of “free” water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an “impervious” material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were “capped” with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles’* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the “Standard Penetration Resistance” or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles’* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as “N”. The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled “General Notes”.

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in *Giles'* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a “sieve analysis,” which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a “hydrometer analysis” which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

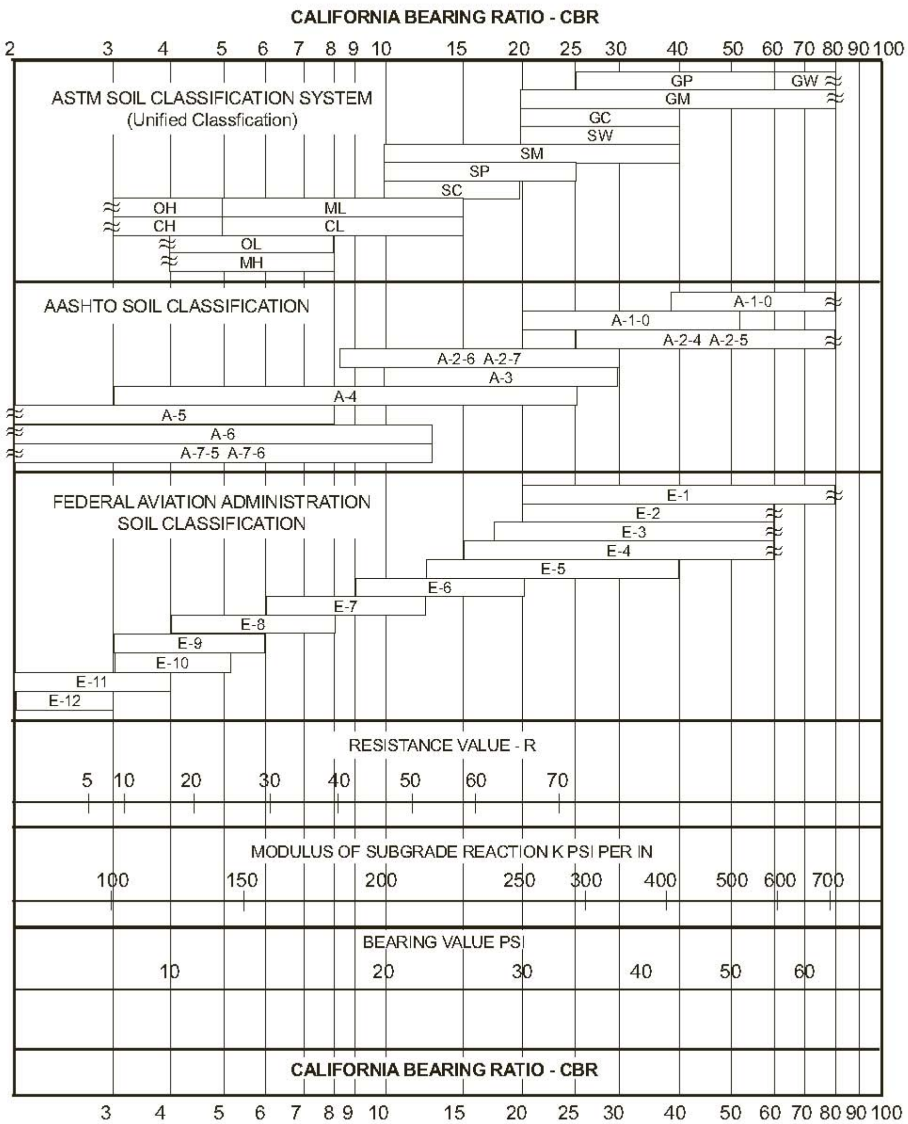
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled “General Notes.”

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING STANDARD PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compaction fill, subgrades and grades shall be (a) underlain by suitable bearing material; (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proof-rolling to detect soil, wet yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated”, and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch-diameter unless specifically approved by an experienced soils engineer. All fill materials must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per the Unified Soil Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than -1 to +3 percent and granular soil ±3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a moisture content 3±1 percent above optimum moisture content to limit further heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavement, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filling, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer’s attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below-grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

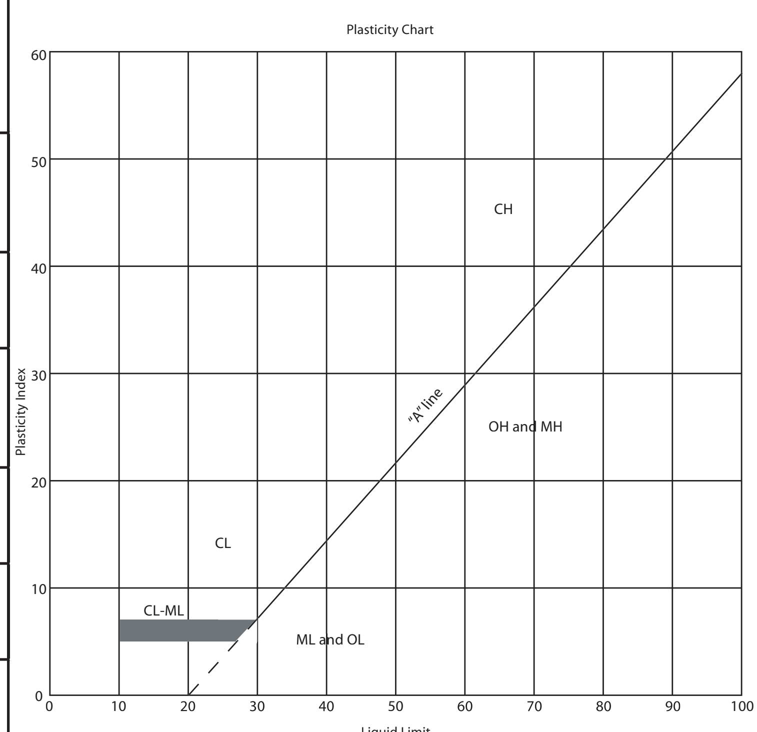
8. Whenever, in the opinion of the soils engineer or the Owner’s Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *									
Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*	*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP; More than 12 percent: GM, GC, SM, SC; 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a] d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a] d / u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt	Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	**BLOWS PER FOOT (N)**	**UNCONFINED COMPRESSIVE STRENGTH (TSF)**
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	**BLOWS PER FOOT (N)**
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	**PI**
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	**PI**
Low	0 - 15
Medium	15 - 25
High	25+

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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Geotechnical, Environmental & Construction Materials Consultants

GILES
ENGINEERING ASSOCIATES, INC.

www.gilesengr.com

ATLANTA, GA
(770) 458-3399

DALLAS, TX
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LOS ANGELES, CA
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MILWAUKEE, WI
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ORLANDO, FL
(407) 321-5356

TAMPA, FL
(813) 283-0096

BALTIMORE/WASHINGTON, D.C.
(410) 636-9320